

10.3—NOTATION

Revise as follows:

ϕ_{qp} = resistance factor for tip resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5) (10.9.3.5.1)
 ϕ_{qs} = resistance factor for ~~shaft~~ side resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5)

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10.5.2.1—General

Revise the 1st Paragraph as follows:

Foundation design at the service limit state shall include:

- Settlements,
- Horizontal movements,
- Overall stability, and
- Total sScour at the design flood.

10.5.2.2—Tolerable Movements and Movement Criteria

Add two paragraphs after the 3rd Paragraph:

Limit eccentricity under Service-I load combination to $B/6$ and $B/4$ when spread footings are founded on soil and rock, respectively.

The permissible (allowable) horizontal load for piles/shafts at abutments shall be evaluated at 0.25 inch pile/shaft top horizontal movement. Horizontal load on the pile from Service-I load combination shall be less than the permissible horizontal load.

C10.5.2.2

Add the following after the last paragraph:

No rotation analysis is necessary when eccentricity under Service-I load combination is limited to $B/6$ and $B/4$ or less for spread footings founded on soil and rock, respectively. Otherwise, it is necessary to establish permissible foundation movement criteria and the corresponding permissible eccentricity limits. When necessary, for bridge abutments such analysis is performed only for eccentricity in the longitudinal direction of the bridge.

The horizontal component of a battered pile's axial load may be subtracted from the total lateral load to determine the applied horizontal or lateral loads on pile foundations.

10.5.3.1—General

Revise the 2nd Paragraph as follows:

The design of all foundations at the strength limit state shall consider:

- Structural resistance and
- Loss of lateral and ~~vertical~~ axial support due to scour at the design flood event.

C10.5.3.1

Revise the 4th Paragraph as follows:

The design ~~event~~ flood for scour is defined in ~~Section 2~~ Article 2.6 and is specified in Article 3.7.5 as applicable at the strength limit state.

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C10.5.4.1

Revise the 1st Paragraph as follows:

Extreme events include ~~the check flood for scour,~~ vessel and vehicle collision, seismic loading, and other site-specific situations that the Engineer determines should be included. Appendix A10 gives additional guidance regarding seismic analysis and design. Scour should be considered with extreme events as per Articles 3.4.1 and 3.7.5.

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10.5.5.2.1 —General

Revise as follows:

Resistance factors for different types of foundation systems at the strength limit state shall be taken as specified in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5, ~~unless regionally specific values or substantial successful experience is available to justify higher values.~~

C10.5.5.2.1

Revise as follows:

~~Regionally specific values should be determined based on substantial statistical data combined with calibration or substantial successful experience to justify higher values.~~ Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that have not been mitigated through conservative selection of design parameters. When a single pile or drilled shaft supports a bridge column, reduction of the resistance factors in Articles 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5 should be considered.

Certain resistance factors in Articles 10.5.5.2.2, 10.5.5.2.3 and 10.5.5.2.4 are presented as a function of soil type, e.g., cohesionless or cohesive sand or clay. Many ~~N~~naturally occurring soils do not fall neatly into these two classifications. In general, the terms "sand" and "cohesionless soil" or "sand" may be connoted to mean drained conditions during loading, while "clay" or "cohesive soil" or "clay" implies undrained conditions in the short-term. For other or intermediate soil classifications, such as clayey sand or silts or gravels, the designer should choose, depending on the load case under consideration, whether the resistance provided by the soil in the short-term will be a drained ~~or~~ undrained, or a combination of the two strengths ~~strength~~, and select the method of computing resistance and associated resistance factor accordingly.

In general, resistance factors for bridge and other structure design have been derived to achieve a reliability index, β , of 3.5, an approximate probability of failure, P_f , of 1 in 5,000. However, past geotechnical design practice has resulted in an effective reliability index, β , of 3.0, or an approximate probability of a failure of 1 in 1,000, for foundations in general, and for highly redundant systems, such as pile groups, an approximate reliability index, β , of 2.3, an approximate probability of failure of 1 in 100 (Zhang et al., 2001; Paikowsky et al., 2004; Allen, 2005). ~~If the resistance factors provided in this article are adjusted to account for regional practices using statistical data and calibration, they should be developed using the β values provided above, with consideration given to the redundancy in the foundation system.~~

For bearing resistance, lateral resistance, and uplift calculations, the focus of the calculation is on the individual foundation element, e.g., a single pile or drilled shaft. Since these foundation elements are usually part of a foundation unit that contains multiple elements, failure of one of these foundation elements usually does not cause the entire foundation unit to reach failure, i.e., due to load sharing and overall unit is usually more, and in many cases considerably

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more, than the reliability of the individual foundation element. Hence, a lower reliability can be successfully used for redundant foundations than is typically the case for the superstructure.

Note that not all of the resistance factors provided in this article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where adequate quantity and/or quality of data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g. the Caltrans Bridge Design Specifications (2000), dated November 2003, AASHTO Standard Specifications for Highway Bridges (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of some of the resistance factors for foundations provided in this article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

The foundation resistance after scour due to the design flood shall provide adequate foundation factored resistance using the resistance factors given in this article.

10.5.5.2.2—Spread Footings

Revise as follows:

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, ~~with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.~~

Revise Table 10.5.5.2.2-1 as follows:

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Nominal Resistance	Resistance Determination Method/Soil/Condition		Resistance Factor
<u>Bearing Resistance in Compression</u>	ϕ_b	Theoretical method (Munfakh et al., 2001), in clay <u>cohesive soils</u>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	0.45
		Semi-Empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	ϕ_τ	Precast concrete placed on sand	0.90
		Cast-in-place concrete on sand	0.80
		Cast-in-place or pre-cast concrete on clay	0.85
		Soil on soil	0.90
	ϕ_{ep}	Passive earth pressure component of sliding resistance	0.50

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*10.5.5.2.3— Driven Piles**C10.5.5.2.3*

Delete the entire article and replace with the following:

Delete the entire commentary and replace with the following:

Resistance factors for driven piles shall be selected from Table 10.5.5.2.3-1.

The resistance factors in Table 10.5.5.2.3-1 are based on engineering judgment, and past WSD and Load Factored Design (LFD) practices.

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Replace Table 10.5.5.2.3-1 as follows:

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles

<u>Nominal Resistance</u>	<u>Resistance Determination Method/Conditions</u>	<u>Resistance Factor</u>	
<u>Axial Compression or Tension</u>	<u>All resistance determination methods, all soils and rock</u>	$\phi_{stat}, \phi_{dyn}, \phi_{qp}, \phi_{qs}, \phi_{bl}, \phi_{up}, \phi_{ug}, \phi_{load}$	<u>0.70</u>
<u>Lateral or Horizontal Resistance</u>	<u>All soils and rock</u>		<u>1.0</u>
<u>Pile Drivability Analysis</u>	<u>Steel Piles</u>	ϕ_{da}	<u>See the provisions of Article 6.5.4.2</u>
	<u>Concrete Piles</u>		<u>See the provisions of Article 5.5.4.2.1</u>
	<u>Timber Piles</u>		<u>See the provisions of Articles 8.5.2.2</u>
	<u>In all three Articles identified above, use ϕ identified as “resistance during pile driving”</u>		
<u>Structural Limit States</u>	<u>Steel Piles</u>	<u>See the provisions of Article 6.5.4.2</u>	
	<u>Concrete Piles</u>	<u>See the provisions of Article 5.5.4.2.1</u>	
	<u>Timber Piles</u>	<u>See the provisions of Articles 8.5.2.2 and 8.5.2.3</u>	

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10.5.5.2.4—Drilled Shafts

Delete the entire article and replace with the following:

Resistance factors for drilled shafts shall be selected from Table 10.5.5.2.4-1.

C10.5.5.2.4

Delete the entire commentary and replace with the following:

The resistance factors in Table 10.5.5.2.4-1 are based on engineering judgment, and past WSD and LFD practices.

The maximum value of the resistance factors in Table 10.5.5.2.4-1 are based on an assumed normal level of field quality control during shaft construction. If a normal level of quality control can not be assured, lower resistance factors should be used.

The mobilization of drilled shaft tip resistance is uncertain as it depends on many factors including soil types, groundwater conditions, drilling and hole support methods, the degree of quality control on the drilling slurry and the base cleanout, etc. Allowance of the full effectiveness of the tip resistance should be permitted only when cleaning of the bottom of the drilled shaft hole is specified and can be acceptably completed before concrete placement.

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Replace Table with the following:

Table 10.5.5.2.4-1—Geotechnical Resistance Factors for ~~Geotechnical~~ Resistance of Drilled Shafts

<u>Nominal Resistance</u>	<u>Resistance Determination Method/Conditions</u>	<u>Resistance Factor</u>	
<u>Axial Compression and Tension or Uplift</u>	<u>All soils, rock and IGM</u> <u>All calculation methods</u>	$\phi_{stat}, \phi_{up}, \phi_{bl}, \phi_{ug},$ $\phi_{load}, \phi_{upload}, \phi_{qs}$	<u>0.70</u>
<u>Axial Compression</u>	<u>All soils, rock and IGM</u> <u>All calculation methods</u>	ϕ_{gp}	<u>0.50</u>
<u>Lateral Geotechnical Resistance</u>	<u>All soils, rock and IGM</u> <u>All calculation methods</u>		<u>1.0</u>

10.5.5.3.2—Scour

Delete the entire article.

C10.5.5.3.2

Revise the 1st Paragraph as follows:

~~The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Paikowsky et al. (2004) regarding bias values for pile resistance prediction methods. See Commentary to Article 3.4.1, Extreme Events, and Article 3.7.5.~~

*10.5.5.3.3—Other Extreme Event Limit States**C10.5.5.3.3*

Revise the 1st Paragraph as follows:

Resistance factors for extreme event limit states, including the design of foundations to resist earthquake, blast, ice, vehicle or vessel impact loads, shall be taken as 1.0. ~~For the uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.~~

Delete the entire commentary:

~~The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see Article C10.5.5.2.3).~~

10.6.1.1—General**C10.6.1.1**

Revise the 1st Paragraph as follows:

Provisions of this article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and others substructure and superstructure elements. Special attention shall be given to footings on fill, to make sure that the quality of the fill placed below the footing is well controlled and of adequate quality in terms of shear strength, swell or expansion potential and compressibility to support the footing loads.

Revise the commentary as follows:

Spread footing should not be used on soil or rock conditions that are determined to be expansive, collapsible, or too soft or weak to support the design loads, without excessive movements, or loss of stability.

10.6.1.3—Effective Footing Dimensions

Revise as follows:

For eccentrically loaded footings, a reduced effective area, $B' \times L'$, within the confines of the physical footing shall be used in geotechnical design for settlement and bearing resistance. The point of load application shall be at the centroid of the reduced effective area.

The reduced dimensions for an eccentrically rectangular footing shall be taken as:

$$B' = B - 2e_B \quad (10.6.1.3-1)$$

$$L' = L - 2e_L$$

Where,

$e_B = \underline{M_L/V} \equiv$ Eccentricity parallel to dimension B (ft)

$e_L = \underline{M_B/V} \equiv$ Eccentricity parallel to dimension L (ft)

$\underline{M_B} \equiv$ Factored moment about the central axis along dimension B (kip-ft)

$\underline{M_L} \equiv$ Factored moment about the central axial along dimension L (kip-ft)

$\underline{V} \equiv$ Factored vertical load (kips)

10.6.1.4—Bearing Stress Distributions

Revise 1st Paragraph as follows:

When proportioning footings dimensions to meet settlement and bearing resistance requirements at all applicable limit states, the distribution of bearing stress ~~on the effective area~~ shall be assumed as:

- Uniform over the effective area for footing on soils, or

C10.6.1.3

Add the following reference:

For additional guidance, see Munfakh (2001) and Article 10.6.3.2.

10.6.1.6—Groundwater

Modify the last paragraph as follows:

The influences of groundwater ~~table~~ on the bearing resistance of soil or rock, the expansion and collapse potential of soil or rock, and on the settlements of the structure should be considered. In cases where seepage forces are present, they should also be included in the analyses.

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C10.6.2.4.1

Insert the following after the last paragraph:

For eccentrically loaded footings on soils, replace L and B in these specifications with the effective dimensions L' and B' , respectively.

10.6.2.4.2—Settlement of Footing on Cohesionless
Soils

C10.6.2.4.2

Revise the 3rd Paragraph as follows:

The elastic half-space method assumes the footing is flexible and is supported on a homogeneous soil of infinite depth. The elastic settlement of spread footings, in feet, by the elastic half-space method shall be estimated as:

Modify the 6th Paragraph as follows:

~~The stress distribution used to calculate elastic settlement assume the footing is flexible and supported on a homogeneous soil of infinite depth. In Table 10.6.2.4.2-1, the β_z values for the flexible foundations correspond to the average settlement. The elastic settlement below a flexible footing varies from a maximum near the center to a minimum at the edge equal to about 50 percent and 64 percent of the maximum for rectangular and circular footing, respectively. For low values of L/B ratio, the average settlement for flexible footing is about 85 percent of the maximum settlement near the center.~~ The settlement profile for rigid footings is assumed to be uniform across the width of the footing.

10.6.2.4.2—Settlement of Footing on Cohesionless
Soils

C10.6.2.4.2

Revise the last sentence in the last paragraph as follows:

Modify the last sentence of the 8th Paragraph as follows:

In Figure 10.6.2.4.2-1, $N_1 A'$ shall be taken as $(N_1)_{60}$ ~~A'_{60}~~ , Standard Penetration Resistance, N (blows/ft), corrected for hammer energy efficiency and overburden pressure as specified in Article 10.4.6.2.4.

Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent ~~footings~~ foundations.

*10.6.2.4.3—Settlement of Footings on Cohesive
Soils*

Insert the following after the 1st Paragraph:

Immediate or elastic settlement of footings founded
on cohesive soils can be estimated using
Eq.10.6.2.4.2-1 with the appropriate value of the soil
modulus.

*10.6.2.4.3—Settlement of Footings on Cohesive
Soils*

Insert the following under Figure 10.6.2.4.3-3:

For eccentrically loaded footings, replace B/H_c with B'/H_c in Figure 10.6.2.4.3-3.

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C10.6.3.1.2e

Replace H with H_{s2} in Eqs. C10.6.3.1.2e-5 and C10.6.3.1.2e-6 of commentary.

Revise equations as follows:

- For circular or square footings:

$$\beta_m = \frac{B}{4H} \quad \beta_m = \frac{B}{4H_{s2}} \quad (\text{C10.6.3.1.2e-5})$$

$$N_c^* = 6.17$$

- For strip footings:

$$\beta_m = \frac{B}{2H} \quad \beta_m = \frac{B}{2H_{s2}} \quad (\text{C10.6.3.1.2e-6})$$

$$N_c^* = 5.14$$

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10.6.3.1.2e—Two-Layered Soil System in
Undrained Loading

Replace H with H_{s2} in Figure 10.6.3.1.2e-2.

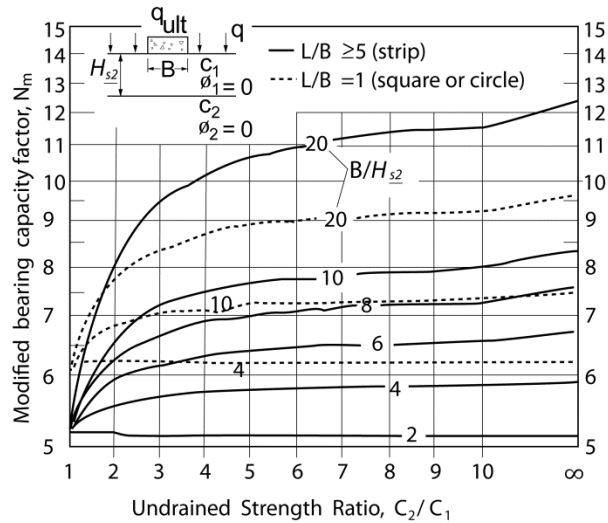


Figure 10.6.3.1.2e-2—Modified Bearing Factor for Two-Layer Cohesive Soil with Weaker Soil Overlying Stronger Soil (EPRI, 1983)

10.6.3.1.2f—Two-Layered Soil System in Drained Loading

Replace H with H_{s2} in Eq. 10.6.3.1.2f-1.

Revise equation as follows:

$$q_n = \frac{\left[q_2 + \left(\frac{1}{K} \right) c'_1 \cot \phi'_1 \right] e^{2 \left[1 + \left(\frac{B}{L} \right) K \tan \phi'_1 \left(\frac{H}{B} \right) \right]} - \left(\frac{1}{K} \right) c'_1 \cot \phi'_1}{\left(\frac{1}{K} \right) c'_1 \cot \phi'_1} \quad (10.6.3.1.2f-1)$$

Revise title as follows:

10.6.3.1.3—Semiempirical Procedures for Cohesionless Soils

C10.6.3.1.2f

Replace H with H_{s2} in Eq. C10.6.3.1.2f-1 of the commentary.

Revise equation as follows:

$$q_n = q_2 e^{\frac{0.67 \left[1 + \left(\frac{B}{L} \right) \right] H}{B}}$$

$$q_n = q_2 e^{\frac{0.67 \left[1 + \left(\frac{B}{L} \right) \right] H_{s2}}{B}} \quad (C10.6.3.1.2f-1)$$

C10.6.3.1.3

Add the following to the end of article:

It is recommended that the SPT based method not be used.

C10.6.3.2.1

Revise as follows:

The design of spread footings bearing on rock is frequently controlled by either overall stability, i.e., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability at the service limit state and size the footing based on eccentricity requirements at the service ~~strength~~ limit state before checking nominal bearing resistance at both ~~the service and~~ strength and extreme event limit states.

Revise title as follows:

10.6.3.2.4—Plate Load Test

Revise as follows:

Where appropriate, plate load tests may be performed to determine the nominal bearing resistance of foundations on rock.

10.6.3.3—Eccentric Load Limitations

Revise as follows:

~~The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:~~

- ~~• One third of the corresponding footing dimension, B or L , for footings on soils, or 0.45 of the corresponding footing dimensions B or L , for footings on rock.~~

The factored nominal bearing resistance of the effective footing area shall be equal to or greater than the factored bearing stress.

C10.6.3.3

Revise as follows:

~~A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of $B/3$ were comparable to those of ASD with an eccentricity of $B/6$. For foundations on rock, to obtain equivalence with ASD specifications, a maximum eccentricity of $B/2$ would be needed for LRFD. However, a slightly smaller maximum eccentricity has been specified to account for the potential unknown future loading that could push the resultant outside the footing dimensions.~~

Excessive differential contact stress due to eccentric loading can cause a footing to rotate excessively leading to failure. To prevent rotation, the footing must be sized to provide adequate factored bearing resistance under the vertical eccentric load that causes the highest bearing stress. As any increase in eccentricity will reduce the effective area of the footing (on soil), or will increase the maximum bearing stress (on rock), bearing resistance check for all potential factored load combinations will ensure that eccentricity will not be excessive

10.6.3.4—Failure by Sliding

Revise Figure 10.6.3.4-1 as follows:

Replace Q_τ with R_τ

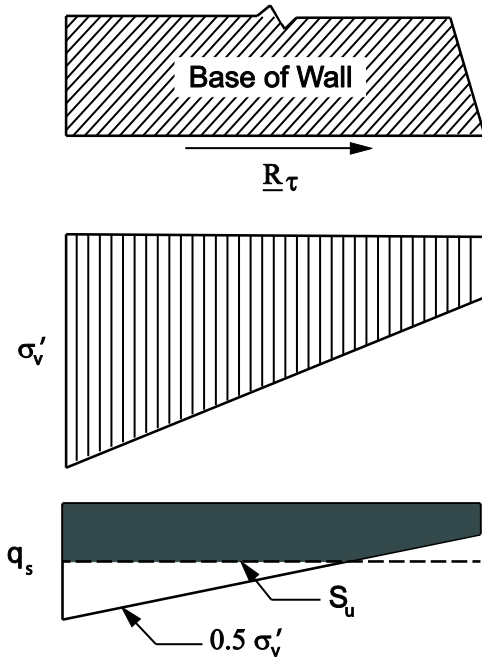


Figure 10.6.3.4-1—Procedure for Estimating Nominal Sliding Resistance for Walls on Clay

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**10.7.1.2—Minimum Pile Spacing, Clearance,
and Embedment into Cap**

Revise the 1st Paragraph as follows:

Center-to-center spacing should not be less than 30.0 in. or 2.5 36.0 in. and 2.0 pile diameters. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 in. and 0.5 pile diameters.

Revise the 2nd Paragraph as follows:

~~The tops of piles shall project at least 12.0 in. into the pile cap after all damaged material has been removed.~~ If the pile is attached to the cap by embedded bars or strands, the pile shall extend no less than 6.0 in. 3.0 in. into the cap for concrete piles and 5 in. into the cap for steel piles.

10. 7.1.4—Battered Piles

Add the following at the end of the article:

In general, battered piles should not be used for foundations of bents and piers.

10. 7.1.5—Pile Design Requirements

Revise as follows:

Pile design shall address the following issues as appropriate:

- Pile cut off elevation, ~~Nominal bearing resistance to be specified in the contract~~, type of pile, and size and layout of pile group required to provide adequate support, with consideration of subsurface conditions, loading, constructability and how nominal bearing pile resistance will be determined in the field.
- Group interaction.
- Pile quantity estimation from estimated pile penetration ~~required~~ to meet nominal axial resistance and other design requirements.
- ~~Minimum pile penetration necessary to satisfy the requirements caused by u~~Uplift, lateral loads, scour, downdrag, settlement, liquefaction, lateral spreading loads, and other seismic conditions.
- Foundation deflection to meet the established movement and associated structure performance criteria.
- Minimum pile penetration necessary to satisfy the requirements caused by settlement, uplift and lateral loads.
- Pile foundation nominal structural resistance.
- Pile foundation buckling and lateral stability.
- Pile drivability to confirm that acceptable driving stresses and blow counts can be achieved at the nominal bearing resistance, and at the estimated resistance to reach the minimum tip elevation, if a minimum tip elevation is required, with an available driving system.
- Long-term durability of the pile in service, i.e., corrosion and deterioration.

C10.7.1.6.2

Revise the 1st and 2nd Paragraphs as follows:

Static downdrag does not affect the ultimate geotechnical capacity or nominal resistance of the pile foundations. It acts to increase pile settlement, and the load on the pile or pile group and the cap. Downdrag occurs when settlement of soils along the side of the piles results in downward movement of the soil relative to the pile. See commentary to Article C3.11.8.

In the case of friction piles with limited tip resistance, the downdrag load can exceed the geotechnical resistance of the pile, causing the pile to move downward enough to allow service limit state criteria for the structure to be exceeded. Where pile settlement is not limited by nominal bearing resistance below the downdrag zone, service limit state tolerances ~~will~~ may govern the geotechnical design.

10.7.2.2—Tolerable Movements

Revise as follows:

The provisions of Article 10.5.2.4 2 shall apply.

10.7.2.3—Settlement**C10.7.2.2**

Revise as follows:

See Article C10.5.2.4 2.

C10.7.2.3

Add the following:

Since most piles are placed as groups, estimation of settlement is more commonly performed for pile groups than a single pile. The equivalent footing or the equivalent pier methods may be used to estimate pile group settlement.

The short-term load-settlement relationship for a single pile can be estimated by using procedures provided by Poulos and Davis (1974), Randolph and Wroth (1978), and empirical load-transfer relationship or skin friction t - z curves and base resistance q - z curves. Load transfer relationships presented in API (2003) and in Article 10.8.2.2.2 can be used. Long-term or consolidation settlement for a single pile may be estimated according to the equivalent footing or pier method.

Revise title as follows:

10.7.2.3.2 Pile Groups Settlement ~~in Cohesive Soil~~

Revise the 1st Paragraph as follows:

Shallow foundation settlement estimation procedures in Article 10.6.2.4 shall be used to estimate the settlement of a pile group, using the equivalent footing location specified in Figure ~~10.7.2.3.1 1-1~~ 10.7.2.3.1-1 or Figure 10.7.2.3.1-2.

Revise the 2nd Paragraph as follows:

The settlement of pile groups in homogeneous cohesionless soils deposits not underlain by more compressible soil at deeper depth may be taken as:

q = net foundation pressure ~~applied at $2D_b/3$ as shown in Figure 10.7.2.3.1 1~~; this pressure is equal to the applied load at the top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles. For friction piles, this pressure is applied at two-thirds of the pile embedment depth, D_b , in the cohesionless bearing layer. For a group of end bearing piles, this pressure is applied at the elevation of the pile tip. (ksf)

D_b = depth of embedment of piles in the cohesionless layer that provides support, as specified in Figure 10.7.2.3.1 1 (ft)

Revise the 4th Paragraph as follows:

The corrected *SPT* blow count or the static cone tip resistance should be averaged over a depth equal to the pile group width B below the equivalent footing. ~~The *SPT* and *CPT* methods (Eqs. 10.7.2.3.2 1 and 10.7.2.3.2 2) shall only be considered applicable to the distributions shown in Figure 10.7.2.3.1 1b and Figure 10.7.2.3.1 2.~~

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**10.7.2.4—Horizontal Pile Foundation
 Movement**

C10.7.2.4

Revise Table as follows:

Table 10.7.2.4-1 Pile P-Multipliers, P_m for Multiple Row Shading (average from Hannigan et al., 2005).

Pile CTC spacing (in the Direction of Loading)	P-Multipliers, P_m		
	Row 1	Row 2	Row 3
<u>2.0B</u>	<u>0.60</u>	<u>0.35</u>	<u>0.25</u>
<u>3.0B</u>	<u>0.75</u> 0.8	<u>0.55</u> 0.4	<u>0.40</u> 0.3
<u>5.0B</u>	1.0	0.85	<u>0.70</u>
<u>7.0B</u>	<u>1.0</u>	<u>1.0</u>	<u>0.90</u>

Revise the 7th Paragraph as follows:

Loading direction and spacing shall be taken as defined in Figure 10.7.2.4-1. A P-multiplier of 1.0 shall be used for pile CTC spacing of 8B or greater. If the loading direction for a single row of piles is perpendicular to the row (bottom detail in the Figure), a P-multiplier group reduction factor of less than 1.0 shall only be used if the pile spacing is 5.4B or less; i.e., a P_m of 0.8 for a spacing of 3B, as shown in Figure 10.7.2.4-1. A P-multiplier of 0.80, 0.90 and 1.0 shall be used for pile spacing of 2.5B, 3B and 4B, respectively.

Revise the 6th Paragraph as follows:

The multipliers on the pile rows are a topic of current research and may change in the future. Values from recent research have been ~~tabulated by~~ compiled from Reese and Van Impe (2000), Caltrans (2003), Hannigan et al. (2006), and Rollins et al. (2006).

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10.7.2.5—Settlement Due to Downdrag

Delete the 1st and 2nd Paragraphs and add the following:

The effects of downdrag, if present, shall be considered when estimating pile settlement under service limit state.

10.7.3.1—General

Revise as follows:

For strength limit state design, the following shall be determined:

- Loads and performance requirements;
- Pile type, dimensions, and nominal bearing resistance;
- Size and configuration of the pile group to provide adequate foundation support;
- The specified pile tip elevation ~~Estimated pile length~~ to be used in the construction contract document to provide a basis for bidding;
- A minimum pile penetration, if required, for the particular site conditions and loading, determined based on the maximum (deepest) depth needed to meet all of the applicable requirements identified in Article 10.7.6;
- The maximum driving resistance expected in order to reach the specified tip elevation ~~minimum pile penetration required, if applicable~~, including any soil/pile side resistance that will not contribute to the long-term nominal bearing resistance of the pile, e.g., surficial soft or loose soil layers, soil contributing to downdrag, or soil that will be removed by scour;
- The drivability of the selected pile to the specified tip elevation ~~achieve the required nominal axial resistance or minimum penetration~~ with acceptable driving stresses at a satisfactory blow count per unit length of penetration; and
- The nominal structural resistance of the pile and/or pile group.

C10.7.2.5—Settlement Due to Downdrag

Delete the 1st and 2nd Paragraphs and add the following:

Guidance to estimate the pile settlement considering the effects of downdrag is provided in Meyerhof (1976), Briaud and Tucker (1997), and Hennigan et al (2005).

C10.7.3.1

Revise the 1st Paragraph as follows:

A minimum pile penetration should only be specified if needed to ensure that uplift, lateral stability, depth to resist downdrag, depth to satisfy scour concerns, and depth for structural lateral resistance are met for the strength limit state, in addition to similar requirements for the service and extreme event limit states. See Article 10.7.6 for additional details. ~~Assuming static load tests, dynamic methods, e.g., dynamic test with signal matching, wave equation, pile formulae, etc., are used during pile installation to establish when the nominal bearing resistance has been met, a minimum pile penetration should not be used to ensure that the required nominal pile bearing, i.e., compression, resistance is obtained.~~

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Revise the title as follows:

10.7.3.3—Pile Length Estimates for Contract Documents

Revise as follows:

Subsurface geotechnical information combined with static analysis methods (Article 10.7.3.8.6), preconstruction test pile programs (Article 10.7.9), and/or pile load tests (Article 10.7.3.8.2) shall be used to estimate the depth of penetration required to achieve the desired nominal bearing for establishment of contract pile quantities. Local experience shall also be considered when making pile quantity estimates, both to select an estimation method and to assess the potential prediction bias for the method used to account for any tendency to over-predict or under-predict pile compressive resistance. If the depth of penetration required to obtain the desired nominal bearing, i.e., compressive, resistance is less than the depth required to meet the provisions of Article 10.7.6, the minimum penetration required per Article 10.7.6 should be used as the basis for the specified tip elevation and estimating contract pile quantities.

C10.7.3.3

Revise the 1st and 2nd Paragraphs as follows:

The estimated pile length necessary to provide the required nominal resistance is determined using a static analysis, local pile driving experience, knowledge of the site subsurface conditions, and/or results from a static pile load test program. The ~~required~~ specified pile tip elevation or length is often defined by the presence of an obvious bearing layer. Local pile driving experience with such a bearing layer should be strongly considered when developing pile quantity estimates.

In variable soils, a program of probe piles across the site ~~is often~~ may be used to determine variable pile order lengths. Probe piles are particularly useful when driving concrete piles. The specified pile tip elevation or length used to estimate quantities for the contract should also consider requirements to satisfy other design considerations, including service and extreme event limit states, as well as minimum pile penetration requirements for lateral stability, uplift, downdrag, scour, group settlement, etc.

Delete the 4th Paragraph.

Revise the 5th Paragraph as follows:

~~The resistance factor for the static analysis method inherently accounts for the bias and uncertainty in the static analysis method. However, local experience may dictate that the penetration depth estimated using this approach be adjusted to reflect that experience. Where piles are driven to a well defined firm bearing stratum, the location of the top of the bearing stratum will dictate the pile length needed, and the Eq. C10.7.3.3-1 is likely not applicable.~~

Delete the 6th Paragraph.

Delete the 7th Paragraph.

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C10.7.3.4.3

Revise the 3rd Paragraph as follows:

If a wave equation or dynamic formula is used to determine the nominal pile bearing resistance on re-strike, care should be used as these approaches require accurate blow count measurement which is inherently difficult at the beginning of redrive (BOR). ~~Furthermore, the resistance factors provided in Table 10.5.5.2.3-1 for driving formulas were developed for end of driving conditions and empirically have been developed based on the assumption that soil setup will occur. See Article C10.5.5.2.3 for additional discussion on this issue.~~

C10.7.3.6

Revise the 1st Paragraph as follows:

The piles will need to be driven to the specified tip elevation and the required nominal bearing resistance plus the side resistance that will be lost due to scour. The nominal resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal bearing resistance plus the magnitude of the side resistance lost as a result of scour, considering the prediction method bias.

Revise the 2nd Paragraph as follows:

The magnitude of skin friction that will be lost due to scour may be estimated by static analysis. Another approach that may be used takes advantage of dynamic measurements. In this case, ~~the static analysis method is used to determine an estimated length. D~~ during the driving of ~~test~~ piles, the side resistance component of the bearing resistance of pile in the scourable material may be determined by a signal matching analysis of the restrike dynamic measurements obtained when the pile tip is below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

C10.7.3.7

Add the following at the end of the article:

Additional guidance to estimate downdrag on single pile and pile groups are provided in ASCE (1993), Briaud and Tucker (1997), and Hennigan et al. (2005).

10.7.3.8.1—General

Revised as follows:

Nominal pile bearing resistance should be field verified during pile installation using static load tests, dynamic tests, wave equation analysis, or dynamic formula. The resistance factor selected for design shall be based on the method used to verify pile axial resistance as specified in Article 10.5.5.2.3. The production piles shall be driven to the specified tip elevation and the minimum blow count determined from the static load test, dynamic test, wave equation, or dynamic formula. ~~and, if required, to a minimum penetration needed for uplift, scour, lateral resistance, or other requirements as specified in Article 10.7.6. If it is determined that static load testing is not feasible and dynamic methods are unsuitable for field verification of nominal bearing resistance, the piles shall be driven to the tip elevation determined from the static analysis, and to meet other limit states as required in Article 10.7.6.~~

C10.7.3.8.1

Revise as follows:

This Article addresses the determination of the nominal bearing (compression) resistance needed to meet strength limit state requirements, using factored loads and factored resistance values. ~~From this design step, the number of piles and pile resistance needed to resist the factored loads applied to the foundation are determined.~~ Both the loads and resistance values are factored as specified in Articles 3.4.1 and 10.5.5.2.3, respectively, for this determination.

In most cases, the nominal resistance of production piles should be controlled by driving to the specified tip elevation and a required blow count. In a few cases, usually piles driven into cohesive soils with little or no toe resistance and very long wait times to achieve the full pile resistance increase due to soil setup, piles may be driven to depth. However, even in those cases, a pile may be selected for testing after a sufficient waiting period, using either a static load test or a dynamic test.

~~In cases where the project is small and the time to achieve soil setup is large compared with the production time to install all the piles, no field testing for the verification of nominal resistance may be acceptable.~~

10.7.3.8.2—Static Load Test

Revise the 1st Paragraph as follows:

If a static pile load test is used to determine the pile axial resistance, the test shall not be performed ~~less than 5 days after the test pile was driven unless approved by the Engineer.~~ prior to completion of the pile set up period as determined by the Engineer. The load test shall follow the procedures specified in ASTM D 1143, and the loading procedure should follow the Quick Load Test Procedure.

C10.7.3.8.2

Revise the Figure C10.7.3.8.2-1 as follows:

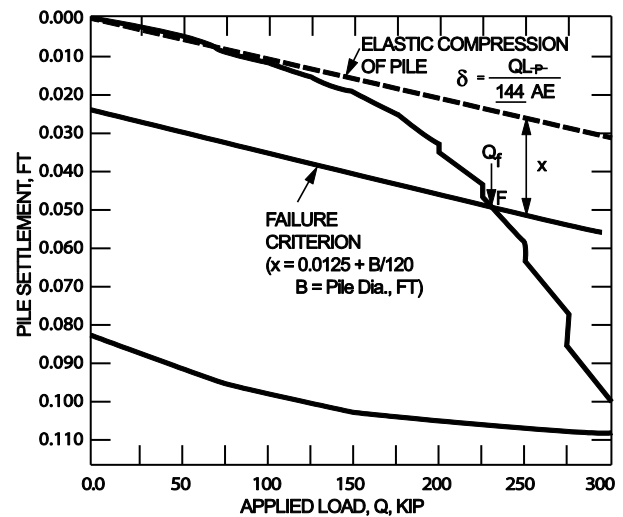


Figure C10.7.3.8.2-1 **Davisson's Alternate Method** for Load Test Interpretation (Cheney and Chassie, 2000, modified after Davisson, 1972)

10.7.3.8.3—Dynamic Testing

Revise the 1st Paragraph as follows:

Dynamic testing shall be performed according to the procedures given in ASTM D 4945. If possible, the dynamic test should be performed as a re-strike test if the Engineer anticipates significant time dependent soil strength change. The pile nominal resistance shall be determined by a signal matching analysis of the dynamic pile test data if the dynamic test is used to establish the driving criteria.

Add the following to the end of the article:

Dynamic testing shall not be used without calibrating to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

C10.7.3.8.3

Revise the 1st Paragraph as follows:

The dynamic test may be used to establish the driving criteria at the beginning of production driving. The minimum number of piles that should be tested are as specified by the Engineer. A signal matching analysis (Rasusche et al., 1972) of the dynamic test data should always be used to determine bearing resistance if a static load test is not performed. See Hannigan et al. (2006) for a description of and procedures to conduct a signal matching analysis. Re-strike testing should be performed if setup or relaxation is anticipated.

10.7.3.8.4—Wave Equation Analysis

C10.7.3.8.4

Add the following to the end of the article:

The wave equation shall not be used without calibrating to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

Revise the 1st Paragraph as follows:

Note that without dynamic test results with signal matching analysis and/or pile load test data (see Articles 10.7.3.8.2 and 10.7.3.8.3), some judgment is required to use the wave equation to predict the pile bearing resistance. Unless experience in similar soils exists, the recommendations of the software provider should be used for dynamic resistance input. Key soil input values that affect the predicted nominal resistance include the soil damping and quake values, the skin friction distribution, e.g., such as could be obtained from a pile bearing static analysis, and the anticipated amount of soil setup or relaxation. The actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, though field observations such as hammer stroke or measured ram velocity can and should be used to improve the accuracy of the wave equation prediction. The reliability of the predicted pile axial nominal resistance can be improved by selecting the key input parameters based on local experience.

10.7.3.8.5—Dynamic Formula

Revise the 1st Paragraph as follows:

If a dynamic formula is used to establish the driving criterion, the following modified FHWA Gates Formula (Eq. 10.7.3.8.5-1) should be used. The nominal pile resistance as measured during driving using this method shall be taken as:

$$R_{ndr} = 1.75 \sqrt{E_d} \log_{10}(10N_b) - 100$$

$$R_{ndr} = [1.83(E_r)^{1/2} \log_{10}(0.83N_b)] - 124 \quad (10.7.3.8.5-1)$$

where:

R_{ndr} = nominal pile resistance measured during pile driving (kips)

E_d = ~~developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke (ft-lb)~~

E_r = Manufacturer's rating for energy developed by the hammer at the observed field drop height (ft-lb)

N_b = Number of hammer blows in the last foot, (maximum value to be used for N is 96) for 1.0 in. of pile permanent set (blows/in. ft).

Delete the 2nd and 3rd Paragraphs.

Revise the 5th Paragraph as follows:

Dynamic formula should not be used when the required nominal resistance exceeds 600 kips or the pile diameter is greater than or equal to 18-in.

C10.7.3.8.5

Delete the 2nd Paragraph as follows:

~~Two dynamic formulas are provided here for the Engineer. If a dynamic formula is used, the FHWA Modified Gates Formula is preferred over the Engineering News Formula. It is discussed further in the Design and Construction of Driven Pile Foundations (Hannigan et al., 2006). Note that the units in the FHWA Gates formula are not consistent. The specified units in Eq. 10.7.3.8.5-1 must be used.~~

Delete the 3rd Paragraph.

~~The Engineering News formula in its traditional form contains a factor of safety of 6.0. For LRFD applications, to produce a nominal resistance, the factor of safety has been removed. As is true of the FHWA Gates formula, the units specified in Eq. 10.7.3.8.5-2 must be used for the Engineering News formula. See Allen (2005, 2007) for additional discussion on the development of the Engineering News formulas and its modification to produce a nominal resistance.~~

Revise the 5th Paragraph as follows:

As the required nominal bearing resistance increases, the reliability of dynamic formulae tends to decrease. The modified FHWA Gates Formula tends to underpredict pile nominal resistance at higher resistances. ~~The Engineering News Formula tends to become unconservative as the nominal pile resistance increases. If other driving formulae are used, the limitation on the maximum driving resistance to be used should be based upon the limits for which the data is considered reliable, and any tendency of the formula to over or under predict pile nominal resistance.~~

C10.7.3.8.6a

Revise as follows:

~~While the most common use of static analysis methods is solely for estimating pile quantities, a static analysis may be used to establish pile installation criteria if dynamic methods are determined to be unsuitable for field verification of nominal bearing resistance. This is applicable on projects where pile quantities are relatively small, pile loads are relatively low, and/or where the setup time is long so that re-strike testing would require an impractical wait period by the Contractor on the site, e.g., soft silts or clays where a large amount of setup is anticipated.~~

The static analysis methods presented in this article should be limited to driven piles 24 in. or less in diameter (length of side for square piles). For steel pipe and cast-in-steel shell (CISS) piles larger than 18 inches in diameter, the static analysis methods from the American Petroleum Institute (API, 2000) publication RP 2A should be used.

~~For use of static analysis methods for contract pile quantity estimation, see Article 10.7.3.3.~~

For open ended pipe piles, the nominal axial resistances should be calculated for both plugged and unplugged conditions. The lower of the two nominal resistances should be used for design.

10.7.3.10—Uplift Resistance of Single Piles

Revise the 1st and 2nd Paragraphs as follows:

Uplift on single piles shall be evaluated when tensile forces are present. The factored nominal tensile resistance of the pile due to soil failure shall be greater than the factored pile loads in uplift or tension.

The nominal uplift resistance of a single pile should be estimated in a manner similar to that for estimating the side friction resistance of piles in compression specified in Article 10.7.3.8.6, and when appropriate, by considering reduction due to the effects of uplift.

Revise the 5th Paragraph as follows:

The static pile uplift load test(s), when performed, should be used to calibrate the static analysis method, i.e., back calculate soil properties, to adjust the calculated uplift resistance for variations in the stratigraphy. The minimum penetration criterion to obtain the desired uplift resistance should be based on the calculated uplift resistance using the static pile load test results-, when available.

C10.7.3.10

Add before the 1st Paragraph as follows:

In general, piles may be considered to resist an intermittent or temporary, but not sustained, uplift by side friction.

Revise the 2nd Paragraph as follows:

~~Note that the resistance factor for uplift already is reduced to 80 percent of the resistance factor for static side friction resistance. Therefore, the side friction resistance estimated based on Article 10.7.3.8.6 does not need to be reduced to account for uplift effects on side friction.~~

See Hannigan et al. (2005) for guidance on the reduction of side friction due to the effects of uplift.

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10.7.3.11—Uplift Resistance of Pile Groups**C10.7.3.11**

Revise the 4th Paragraph as follows:

Add the following to the end:

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be determined using a spread of load of $1H$ in $4V$ from the base of the pile group taken from Figure 10.7.3.11-1. The nominal uplift resistance of the pile group when considered as a block shall be taken as equal to the weight of this soil block. Buoyant unit weights shall be used for soil below the groundwater level. In this case, the resistance factor ϕ_{ug} in Eq. 10.7.3.11-1 shall be taken as equal to 1.0.

In cohesionless soils, the shear resistance around the perimeter of the soil block that will be uplifted is ignored. This results in a conservative estimate of the nominal uplift resistance of the block and justifies the use of a higher resistance factor of 1.0.

Delete the 6th and 7th Paragraphs.

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Revise title as follows:

10.7.5 —Protection Against Corrosion and Deterioration

C10.7.5

Revise the 2nd Paragraph as follows:

As a minimum, the following types of deterioration shall be considered:

- Corrosion of steel pile foundations, particularly in fill soils, ~~low pH soil~~ and marine environment;
- Chloride, ~~Sulfate~~, ~~chloride~~, and acid attack of concrete pile foundations; and
- Decay of timber piles from wetting and drying cycles or from insects or marine borers.

Revise the 3rd Paragraph as follows:

The following soil, water or site conditions ~~should~~ shall be considered as indicative indicators of a potential pile corrosion or deterioration ~~or corrosion situation~~:

- Minimum Resistivity equal to or less than 2,000 1,000 ohm-cm,
- Chloride concentration equal to or greater than 500 ppm,
- Sulfate concentration equal to or greater than 2,000 ppm,
- pH equal to or less than 5.5,
- ~~pH between 5.5 and 8.5 in soils with high organic content,~~
- ~~Sulfate concentration greater than 1,000 ppm,~~
- Landfills and cinder fills,
- ~~Soils subject to m~~ Mine or industrial drainage,
- Suspected chemical wastes, and
- Stray currents
- ~~Areas with a mixture of high resistivity soils and low resistivity high alkaline soils, and~~
- ~~Insects (woof piles)~~

Add the following after the 3rd Paragraph:

Steel piling may be used in corrosive soil and/or water environments provided the following corrosion rates are used to determine a corrosion allowance (sacrificial metal loss):

- 0.001 in. per year for the soil embedment zone.
- 0.004 in. per year for the immersed zone.
- 0.005 in. per year for the splash zone.

The corrosion rates used to determine the corrosion allowance for steel piling shall be doubled for steel H-piling since there are two surfaces for the web and flange that would be exposed to the corrosive environment.

Delete the 4th Paragraph.

Revise the 12th Paragraph as follows:

Epoxy coating of pile reinforcement ~~has been found in some cases to be~~ is useful in resisting corrosion. It is important ~~to ensure~~ that the coating is continuous and free of holidays.

10.8.1.2—Shaft Spacing, Clearance, and Embedment Into Cap

Modify the 1st Paragraph as follows:

The center-to-center spacing of drilled shafts in a group shall be not less than 2.5 times the shaft diameter. If the center to center spacing of drilled shafts is less than 4.0 diameters, the interaction effects between adjacent shafts shall be considered. If the center to center spacing of drilled shafts is less than 6.0 diameters, the sequence of construction should be specified in the contract documents.

Revise title as follows:

10.8.1.3—Shaft Diameter, Concrete Cover, Rebar Spacing, and Enlarged Bases

Revise as follows:

If the shaft is to be manually inspected, the shaft diameter should not be less than 30.0 in. The diameter of columns supported by shafts should be smaller than or equal to the diameter of the drilled shaft.

In order to facilitate construction of the CIDH piles or drilled shafts, the minimum concrete cover to reinforcement (including epoxy coated rebar) shall be as specified in Table 10.8.1.3-1.

Table 10.8.1.3-1—Minimum Concrete Cover for CIDH Piles or Drilled Shafts (to be shown on the plan)

<u>Diameter of the CIDH Pile or Drilled Shaft “D”</u>	<u>Concrete Cover^a</u>
<u>16” and 24” Standard Plan Piles</u>	<u>Refer to the applicable Standard Plans</u>
<u>$24" \leq D \leq 36"$</u>	<u>3”</u>
<u>$42" \leq D \leq 54"$</u>	<u>4”</u>
<u>$60" \leq D < 96"$</u>	<u>5”</u>
<u>96” and larger</u>	<u>6”</u>

^a For shaft capacity calculations, only 3” of cover is assumed effective and shall be used in calculations.

C10.8.1.2

Revise the 1st Paragraph as follows:

Larger spacing may be required to preserve shaft excavation stability or to prevent communication between shafts during excavation and concrete placement. If the center-to-center spacing of drilled shafts is less than 3.0 diameters, the sequence of shaft installation should be specified in the contract documents.

In order to improve concrete flow when constructing drilled shafts, a 5 in. × 5 in. clear window between the horizontal and vertical shaft reinforcing steel shall be maintained, except at the locations of the inspection pipes where the minimum longitudinal reinforcing spacing may be reduced from 5 in. to 3 in.

The maximum center-to-center spacing of longitudinal bars in drilled shafts is limited to 10 in. when the shaft diameter is less than 5 ft and 12 in. for larger shafts. The maximum center-to-center spacing of transverse bars in drilled shafts is limited to 8 in.

When a column is supported on a single enlarged Type II shaft (Caltrans' Seismic Design Criteria 2.2.4), the allowable offset between centerlines of the column (column cage centerline is fixed) and the shaft reinforcement cages shall be limited by the required horizontal clearance between the two cages. The clear distance between the two cages shall be at least 3.5 in. for dry pour and 5 in. for wet pour as shown in Figure 10.8.1.3-1. The offset between centerlines of the shaft cage and the drilled hole, shall be limited to provide minimum concrete cover of 3 in.

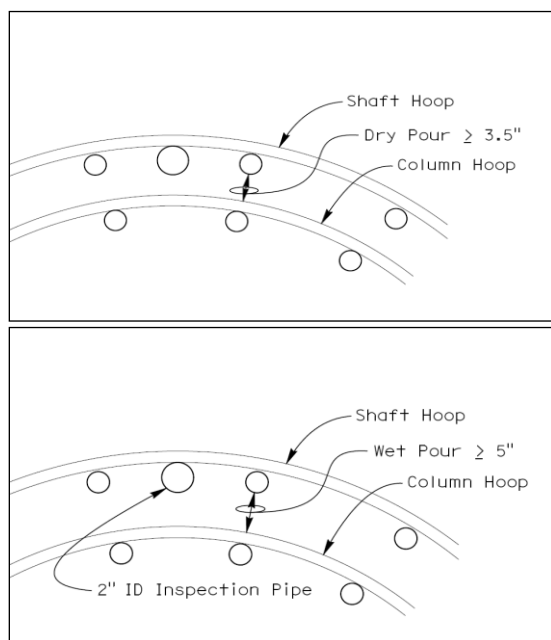


Figure 10.8.1.3-1—Clearance between Column and Shaft Rebar Cages in Enlarged Type II-Shafts

In stiff cohesive soils, an enlarged base (bell, or underream) may be used at the shaft tip to increase the tip bearing area to reduce the unit end bearing pressure or to provide additional resistance to uplift loads.

Where the bottom of the drilled hole is dry, cleaned and inspected prior to concrete placement, the entire base area may be taken as effective in transferring load.

*10.8.2.2.2—Settlement of Single-Drilled Shaft**C10.8.2.2.2*

Add the following to the end of the article.

Superstructure tolerance to support movements shall be verified for the displacements assumed in the geotechnical design of the shaft at the strength limit states.

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10.8.3.5.1c—Tip Resistance

Revise the 1st Paragraph.

For axially loaded shafts in cohesive soil, the net nominal unit tip resistance, q_p , in ksf, by the total stress method as provided in O’Neil and Reese (1999) shall be ~~taken~~ calculated as follows:

If $Z \geq 3D$

$$q_p = N_c S_u \leq 80.0$$

$$q_p = N_c * S_u \quad (10.8.3.5.1c-1)$$

in which:

$$N_c = 6 \left[1 + 0.2 \left(\frac{Z}{D} \right) \right] \leq 9$$

$$N_c = 9 \text{ for } S_u \geq 2 \text{ ksf}$$

$$N_c = \left(\frac{4}{3} \right) \left[1 + \ln(I_r) \right] \text{ for } S_u < 2 \text{ ksf} \quad (10.8.3.5.1c-2)$$

If $Z < 3D$,

$$q_p = \left(\frac{2}{3} \right) \left[1 + \left(\frac{1}{6} \right) \left(\frac{D}{B} \right) \right] N_c * S_u \quad (10.8.3.5.1c-3)$$

where:

D = diameter of drilled shaft (ft)

Z = ~~penetration depth~~ depth of drilled shaft base (ft)

S_u = design undrained shear strength (ksf)

I_r = rigidity index = $(E_s/3S_u)$

E_s = Young’s modulus of soil for undrained loading (ksf)

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10.8.3.6.3—Cohesionless Soil

Revise Table as follows:

Table 10.8.3.6.3-1—Group Reduction Factors for Bearing Resistance of Shafts in Sand

Shaft Group Configuration	Shaft Center-to-Center Spacing	Special Conditions	Reduction Factor for Group Effects, η
Single Row	2D <u>2.5D</u>		0.90 <u>0.95</u>
	3D or more		1.0
Multiple Row	2.5D		0.67
	3D		0.80
	4D or more		1.0
Single and Multiple Rows	2D <u>2.5D</u> or more	Shaft group cap in intimate contact with ground consisting of medium dense or denser soil, and no scour below the shaft cap is anticipated.	1.0
Single and Multiple Rows	2D <u>2.5D</u> or more	Pressure grouting is used along the shaft sides to restore lateral stress losses caused by shaft installation, and the shaft tip is pressure grouted.	1.0

10.8.3.7.2—Uplift Resistance of Single Drilled
Shaft

C10.8.3.7.2

Modify the 1st Paragraph as follows:

Modify the 1st Paragraph as follows:

The uplift resistance of a single straight-sided drilled shaft should be estimated in a manner similar to that for determining side resistance for drilled shafts in compression, as specified in Article 10.8.3.5.3, and, when appropriate, by considering reduction due to effects of uplift.

The side resistance factors for uplift ~~are~~ is lower than that those for axial compression. One reason for this is that drilled shafts in tension unload ~~the~~ the soils, thus reducing the overburden effective stress and hence the uplift side resistance of the drilled shaft. ~~Empirical justification for uplift resistance factors is provided in Article C10.5.5.2.3, and in Allen (2005).~~

Revise title as follows:

**10.9.1.2—Maximum Micropile Diameter and
Minimum Micropile Spacing, Clearance, and
Embedment into Cap**

Revise as follows:

Center-to-center ~~pile~~ spacing of micropiles should not be less than 30.0 in. or 3.0 pile diameters, whichever is greater. Otherwise, the provisions of Article 10.7.1.2 shall apply. The diameter of the micropile drilled hole shall not be greater than 13 in.

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*10.9.3.5.4—Micropile Load Test**C10.9.3.5.4*

Delete the entire article and replace with the following:

Section 49-5 of the *Standard Specifications* and the project special provisions shall supersede Article 10.9.3.5.4.

Delete the entire commentary and replace with the following:

Section 49-5 of the *Standard Specifications* and the project special provisions shall supersede Article C10.9.3.5.4.

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